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1-D CONSOLIDATION OF SOFT CLAY SUBJECTED TO CYCLIC LOADING USING LEM

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Abstract: In this paper, a numerical modification is carried out on the Layer Equation Method (LEM) of El Gendy and Herrmann to be applicable for analyzing 1-D consolidation of soft clay subjected to cyclic loading. The LEM is applicable for multilayered soil system subjected to variable initial stress along depth. The proposed solution is used for normally and over consolidated clays subjected to different types of cyclic loading considering the basis of the method of Toufigh and Ouria. The LEM is incorporated by the authors into the geotechnical software ELPLA and is verified with two verifications. The results of the verifications are close to the references results. The proposed solution is applied for circular storage tanks as a structure subjected to cyclic loading from filling and discharging cycles. An application was held to study the effect of cyclic loading on two zones located at Port-Said city in Egypt using real soil data from real sites. The results of the average degree of consolidation and consolidation settlement versus time are presented for both zones.

Keywords: *1-D Consolidation, cyclic loading, layered soil, water tanks.*

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1.0 Introduction

In geotechnical engineering, subsoils beneath several structures are subjected to complicated time-dependent loading such as cyclic loading paths may result from natural phenomena or human activities. Cyclic loading can be either long-period as filling and discharging in silos and tanks, groundwater level cyclic variation or short-period as wave action, vehicular traffic, blast, and earthquake. Loading and unloading stages in tanks, and vehicle traffic can cause regular time-dependent loading paths. However, time-dependent loading paths caused by blast or earthquake are irregular. If the subsoil subjected to cyclic loading is fine grain saturated soil (as clay and silt), It becomes very important to study the settlement of subsoil because the settlement of saturated fine grain soil is a time- dependent-phenomena. The cyclic conditions will affect the consolidation process, by changing the soil properties which would change the value and rate of settlement at any particular time.

In the case of the normally consolidated (NC) plastic clays under cyclic loadings, consolidation computation will be more complicated and the solution for consolidation partial differential equation will be more difficult too.

There are several methods presented by researchers for over consolidated (OC) elastic clays under cyclic loading. Schiffman (1958) was the first to obtain a general solution of soil consolidation considering loading increase linearly with time. Many other authors studied the consolidation process under different kinds of time-dependent loading such as Wilson and El Gohary (1974), Olson (1977), Baligh and Levadoux (1978), Favaretti and Mazzucato (1994), Xie and Zhuang (2005), Conte and Troncone (2006), Liu and Griffiths (2015) and Qing and Xing (2017).

In recent years, other researches presented formulas for the case of normally consolidated (NC) plastic clays under cyclic loadings. Toufigh and Ouria (2009) presented a semi analytical solution for one dimensional consolidation problem of plastic clays under rectangular cyclic loading considering the effect of the change of the consolidation coefficient of the soil layer. Elastic plastic (Bilinear) model was used to model the plastic behavior of soil under rectangular cyclic loading. Toufigh and Ouria (2010) used their previous model for predicting the further effects of groundwater level oscillation and actual field condition on land subsidence. Abbaspour (2014) performed an experimental study to investigate the consolidation behavior of clay subjected to triangular cyclic loading. Ouria et al. (2015) presented a simplified solution for consolidation of plastic clays under rectangular cyclic loading in disturbed state concept (DSC) framework.

In this study, the Layer Equation Method (LEM) of El Gendy and Herrmann (2014) is developed to study 1-D consolidation of multi-layered soft clay soils subjected to cyclic loading. The LEM are applicable for different types of cyclic loadings such as: rectangular, triangular and trapezoidal. Using the LEM, both normally and over consolidated clays can be analyzed for any stress distribution along the soil depth. Figure 1 shows the soil modeling using the LEM.

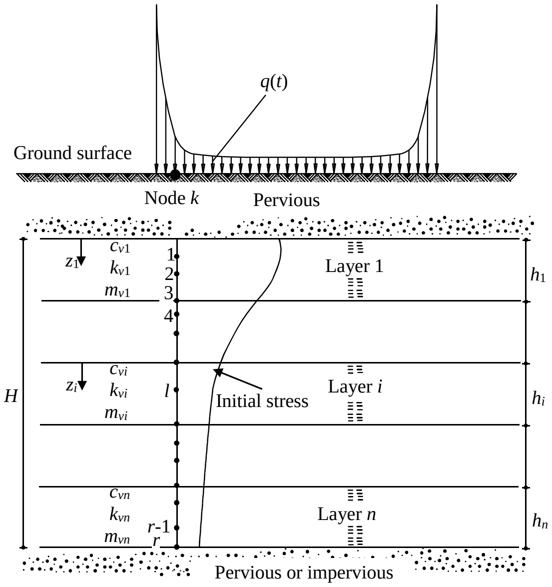

Figure 1 : Soil Model using Layer Equation Method (LEM)

2.0 Methodology

2.1 Behavior of Clay under Cyclic Loading

If the soft clay layer is over consolidated (OC), and the effective stress at the end of every loading phase is smaller than the pre-consolidation pressure, the clay will consolidate and expand with the properties of the over consolidated clay at the route (1-2) and (2-1) as shown in Figure 2. This type of clay is called elastic clay, which remain in the same state under cyclic loading.

Plastic behavior of soft clay soils in a limited stress range under a cyclic loading can be considered using bilinear (elastic – plastic) model that is shown in Figure 3. According to this model, the coefficients of volume change m_v and permeability k_v of plastic clays changes during the loading and unloading phases. Therefore, the coefficient of consolidation C_v is considered as a function of these parameters and changes in each cycle of loading.

At the beginning of loading, soil is in normally consolidated (NC) state and stress path is according to route [1-2], as shown in Figure 3-(a), (b) and (c). During the unloading phase, soil is at OC state and stress path is according to routes [2-3], [5-6] and so on.

After the first full cycle, in the following loading phases, at the beginning, the soil is in OC state until the degree of consolidation reaches the previous maximum degree of consolidation, which is equal to its value at the end of the last loading phase. In Figure 3-(d), Δt_2 is the time portion of the second loading phase in which the soil is in OC state (according to the route [3–4] in Figure 3) and then becomes NC. In order to find the degree of consolidation and settlement in the next cycles, the value of Δt_N for each cycle must be known.

The pre-consolidation pressure σ_{cN} increases by increasing number of cycles and reaches a point where the soil body stays in OC state during the entire loading phases, which is called steady-state condition (SSC). In the steady-state condition the time portion Δt equals the time of the loading phase duration.

In linear analysis of the consolidation process, the coefficient of consolidation C_v is assumed to have only two different values in the state of NC or OC and changes suddenly when the soil changes from NC to OC or vice versa where; $C_{v(\mathrm{NC})} = \beta\ C_{v(\mathrm{OC})}$ as shown in Figure 3-(c). Also, the coefficient of volume change m_v has only two values and changes when the soil changes from NC to OC state or vice versa, where $m_r = \alpha\ m_v$.

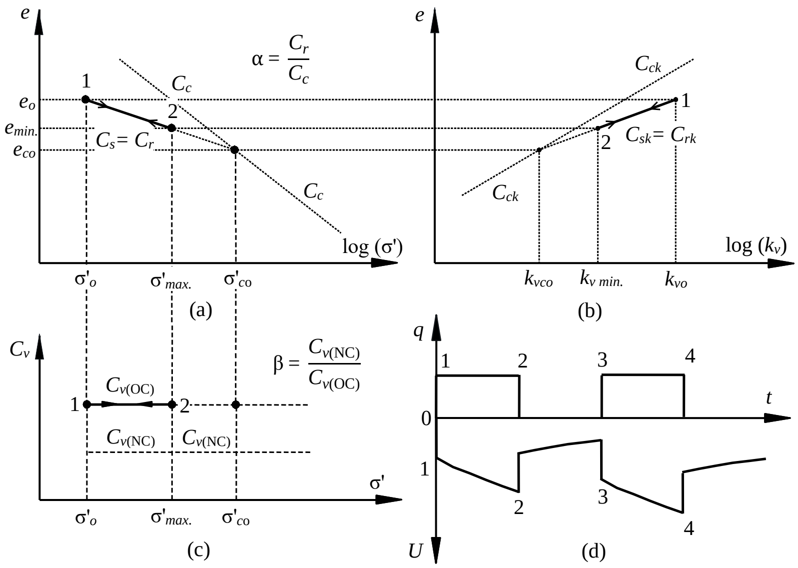

Figure 2 : Behavior of elastic over consolidated clay ($\sigma'_{max} < \sigma'_{co}$): (a) Idealized e-Log(σ') relationship, (b) Idealized e-Log(k_v) relationship, (c) Idealized C_v-σ' relationship, (d) Degree of consolidation caused by rectangular cyclic loading

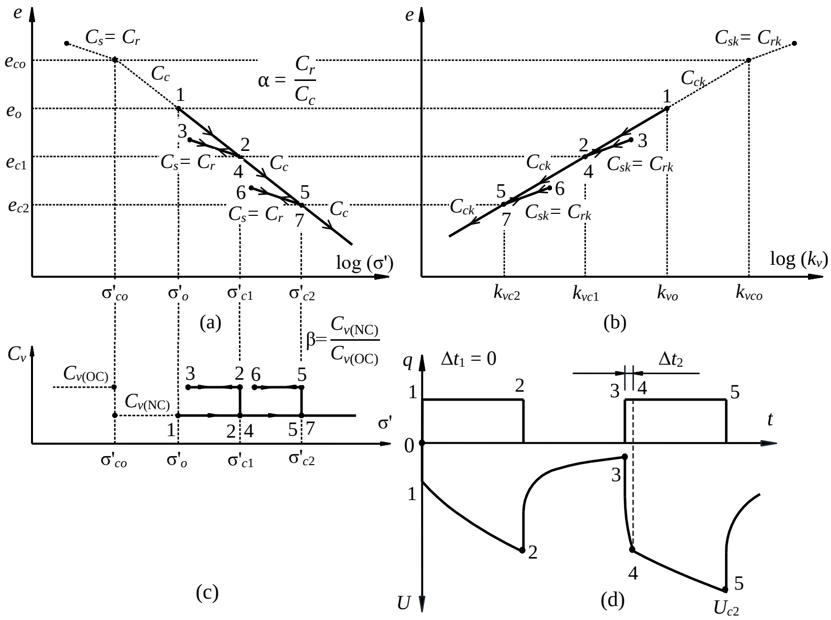

Figure 3 : Behavior of plastic normally consolidated clay ($\sigma'_o > \sigma'_{co}$)): (a) Idealized e-Log(σ') relationship, (b) Idealized e-Log(k_v) relationship, (c) Idealized C_v-σ' relationship, (d) Degree of consolidation caused by rectangular cyclic loading

2.2 Governing Equations for Linear Loading

For a loading constant with time, the excess pore water pressure at any time t at a point k on the soil surface according to *LEM*, is given by:

$$\{u\}_t = [\Phi][E_v]^t[\Phi]^{-1}\{u\}_o \tag{1}$$

where $\{u\}_t$ is the vector of the excess pore water pressure u_i, i = 1 to r, $\{u\}_o$ is the initial excess pore water pressure, $\{\sigma\}_o$ the initial vertical stress vector, $[\Phi]$ is the matrix of basic functions and $[E_v]$ is the diagonal square matrix of the exponential functions. Details of the matrices $[\Phi]$ and $[E_v]$ are listed in *El Gendy and Herrmann* (2014).

In practice, the total load on clay under a structure may be applied linearly over a period of time. In this case, the total load of construction on the surface q_c can be applied gradually over a time t_c as shown in Figure 4.

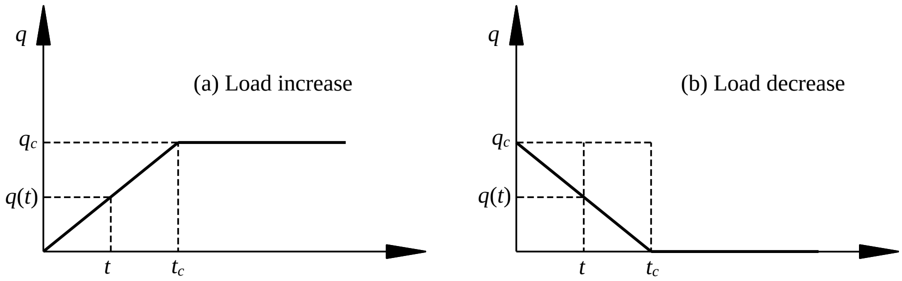

Figure 4 : Linear loading with time

If $t = t_c$ or $t > t_c$, the excess pore water pressure can be found as:

$$\{u\}_t = \pm[\Phi][E_v]^t[D_j]^t[\Phi]^{-1}\{u\}_o \tag{2}$$

where $[D_j]^t$ is a diagonal square matrix for which the element D_j is given by:

$$D_j = \frac{\exp(\omega_j t_c)\text{ -1}}{\omega_j t_c} = \frac{\exp(\lambda_j^2 T_{vc\,1})\text{ -1}}{\lambda_j^2 T_{vc\,1}} \tag{3}$$

where λ_j is the differential equation operator and $T_{vc\,1}$ is the time factor at time t_c for layer number 1 in a multilayered soil system.

In this case $T_{vc\,1}$ can be calculated as:

$$T_{vc\,1} = \frac{C_{v\,1}\, t_c}{h_1^2} \tag{4}$$

where $C_{v\,1}$ is the coefficient of consolidation of layer number 1 in a multilayered soil system in [m^2/ year], h_1 is the thickness of layer number 1, in [m], and ω_j is a factor given by:

$$\omega_j = \frac{C_{v\,1}}{h_1^2}\lambda_j \tag{5}$$

If $t < t_c$, the load is assumed to be proportional to time, then:

$$\frac{q(t)}{q_c} = \frac{t}{t_c} \tag{6}$$

The excess pore water can be expressed as:

$$\{u\}_t = \frac{t}{t_c}[\Phi][E_v]^t[D_j]^t[\Phi]^{-1}\{u\}_o \tag{7}$$

In this case, the diagonal elements of the matrix $[D_j]^t$ are given as:

$$D_j = \frac{\exp(\omega_j t) - 1}{\omega_j t} \tag{8}$$

The excess pore water pressure given in equation (2) would have a positive sign for the case of load increase and it would have a negative sign for the case of load decrease.

Equations (2) and (7) can be combined to express the pore water pressure in a general form as:

$$\{u\}_t = \pm[\Phi][E_v]^t\left[D_j^*\right]^t[\Phi]^{-1}\{u\}_o \tag{9}$$

In this case, the diagonal elements of the matrix $[D_j^*]^t$ are given by:

$$D_j^* = \frac{\exp(\omega_j t) - 1}{\omega_j t_c} = \frac{\exp(\lambda_j^2 T_{v\,1}) - 1}{\lambda_j^2 T_{vc\,1}} \tag{10}$$

where $T_{v\,1}$ is the time factor at time t for layer number 1 in a multilayered soil system, and it can be calculated from equation (4) by replacing t_c with t.

For load increase, the degree of consolidation U_p and U_s at time t can be obtained based on either the stress or the settlement. The degree of consolidation U_p can be written as a function of the stress as:

$$U_p = \frac{\sum_{i=1}^{n}\Delta\sigma(t)_i h_i - \sum_{i=1}^{n}\Delta u_i h_i}{\sum_{i=1}^{n}\Delta\sigma_{oi} h_i} \tag{11}$$

where Δu_i is the average excess pore water pressure at any time factor in that layer, $\Delta\sigma_{oi}$ is the initial average stress in a layer i and $\Delta\sigma(t)_i$ is the average stress in a layer i at time t. In this case, the degree of consolidation U_p can be expressed as a function of the applied current load q as:

$$U_p = \frac{q(t)}{q_c} - \frac{\sum_{i=1}^{n} \Delta u_i h_i}{\sum_{i=1}^{n} \Delta\sigma_{oi} h_i} \tag{12}$$

In the same manner, the degree of consolidation U_s can be written as a function of the settlement as:

$$U_s = \frac{q(t)}{q_c} - \frac{\sum_{i=1}^{n} m_{v\,i} \Delta u_i h_i}{\sum_{i=1}^{n} m_{v\,i} \Delta\sigma_{oi} h_i} \tag{13}$$

where m_{vi} is the coefficient of volume change of a layer i.

For load decrease, both the stress dependent- and the settlement dependent degree of consolidation, U_p and U_s, respectively, can be expressed as:

$$U_p = \left(1 - \frac{q(t)}{q_c}\right) - \frac{\sum_{i=1}^{n} \Delta u_i h_i}{\sum_{i=1}^{n} \Delta\sigma_{oi} h_i} \tag{14}$$

$$U_s = \left(1 - \frac{q(t)}{q_c}\right) - \frac{\sum_{i=1}^{n} m_{v\,i} \Delta u_i h_i}{\sum_{i=1}^{n} m_{v\,i} \Delta\sigma_{oi} h_i} \tag{15}$$

2.3 Governing Equations for Cycling Loading

Figure 5 shows different types of cyclic loading. If the base of the tank is perfectly flexible (when neglecting the tank rigidity), then the contact stress will be equal to the gravity stress exerted by the tank on the underlying soil. The types may be represented any expected cyclic loading shape. In the figure, there are three parameters t_o, α_o and β_o reflect the properties of the loading. The number of cycle is N. The time t_o represents the time length of the subjected load whatever the load geometry is, while the time $\beta_o t_o$ is the total length of the cycle. The parameter α_o represents the load geometry, where α_o = 0 creates a rectangular cyclic loading, α_o = 0.5 creates a triangular cyclic loading and $0<\alpha_o<0.5$ creates a trapezoidal cyclic loading.

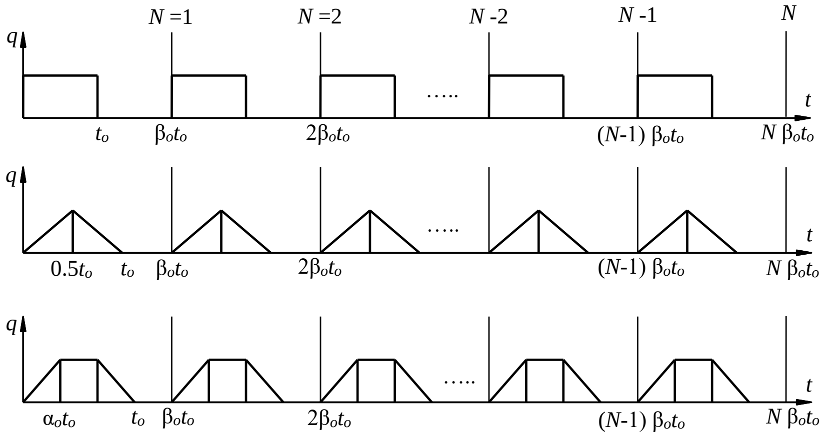

Figure 5 : Types of cyclic loading

The clay is named as plastic clay if the clay changes from a state to another -for example from NC to OC- during the time of the subjected cyclic load. Considering the first cycle of the trapezoidal cyclic load shown in Figure 5. The excess pore water pressure for the loading and constant load phases at any time t, where $0 \leq t \leq (t_o - \alpha_o t_o)$ can be calculated using the following equation:

$$\{u_j\}_t = [\Phi][E_v]^t[D_j]^t[\Phi]^{-1}\{u_j\}_o \tag{16}$$

where $[D_j]^t$ is a square diagonal matrix with elements D_j. The elements D_j are calculated using equations (3) and (10) for trapezoidal and triangular cyclic load, while for rectangular load the elements are $D_j = 1$.

The rest of the first cycle are the unloading and no-loading phases. The excess pore water pressure for these phases at any time t, where $(t_o - \alpha_o t_o) \leq t \leq (\beta_o\ t_o)$ can be calculated using the following equation:

$$\{u_j\}_t = \left[[\Phi][E_v]^t[\Phi]^{-1}\{u_j\}_{(t_o - \alpha_o t_o)}\right] - \left[[\Phi][E_v]^t[D_j]^t[\Phi]^{-1}\{u_j\}_o\right] \tag{17}$$

The constant load phase is considered to continue till time t, while the unloading and no-loading phases -which are the reverse of the loading and constant load phases- are subtracted from the continued values.

Cycle two starts with reloading phase extended to portion time Δt_2, then the phase is changed to loading phase till reaching the time $\{\beta_o\ t_o+(t_o - \alpha_o t_o)\}$as shown in Figure 5. The excess pore water pressure at Δt_2 can be calculated using the following equation:

$$\{u_j\}_{\Delta t_2} = \left[[\Phi][E_v]^{\Delta t_2}[\Phi]^{-1}\{u_j\}_{\beta_o\, t_o}\right] + \left[[\Phi][E_v]^{\Delta t_2}[D_j]^{\Delta t_2}[\Phi]^{-1}\{u_j\}_o\right] \tag{18}$$

Substituting from equation (18) into equations (12) and (13) to find the last maximum degree of consolidation. The value of Δt_2 is difficult to be calculated using closed form solution, so the try and error or iteration methods are successful to get the value of Δt_2.

After that, the excess pore water pressure for the loading and constant load phases at any time t, where $\Delta t_2 \leq t \leq \{\beta_o\, t_o + (t_o - \alpha_o t_o)\}$ can be calculated using the following equation:

$$\{u_j\}_t = \left[[\Phi][E_v]^t[\Phi]^{-1}\{u_j\}_{\beta_o\, t_o}\right] + \left[[\Phi][E_v]^t[D_j]^t[\Phi]^{-1}\{u_j\}_o\right] \tag{19}$$

where the time factor T_v in the square matrix $[E_v]^t$ is calculated according to *Toufigh* and *Ouria* (2009) and using the following equation:

$$T'_v = \frac{\Delta T_N}{\beta} + (T_v - \Delta T_N) \tag{20}$$

Where T'_v is the time factor of time t in the virtual time scale, ΔT_N is the time factor of time portion Δt_N for cycle number N in the real-time scale, β is the virtual time factor and equal $C_{v\,(NC)}/\, C_{v\,(OC)}$ and T_v is the time factor of time t in the real-time scale, [-]. The rest of the second cycle are the unloading and no-loading phases. The excess pore water pressure for these phases at any time t, where $\{\beta_o\, t_o+(t_o - \alpha_o t_o)\} \leq t \leq (2\beta_o\, t_o)$ can be calculated using the following equation:

$$\{u_j\}_t = \left[[\Phi][E_v]^t[\Phi]^{-1}\{u_j\}_{\{\beta_o\, t_o+(t_o - \alpha_o t_o)\}}\right] - \left[[\Phi][E_v]^t[D_j]^t[\Phi]^{-1}\{u_j\}_o\right] \tag{21}$$

For the general case of cyclic loading, the excess pore water pressure for loading and constant load phases can be determined using the following equation:

$$\{u_j\}_t = \left[[\Phi][E_v]^t[\Phi]^{-1}\{u_j\}_{(N-1)\beta_o\, t_o}\right] + \left[[\Phi][E_v]^t[D_j]^t[\Phi]^{-1}\{u_j\}_o\right] \tag{22}$$

And, the excess pore water pressure for unloading and no-loading phases can be determined using the following equation:

$$\{u_j\}_t = \left[[\Phi][E_v]^t[\Phi]^{-1}\{u_j\}_{\{(N-1)\beta_o\, t_o+(t_o - \alpha_o t_o)\}}\right] - \left[[\Phi][E_v]^t[D_j]^t[\Phi]^{-1}\{u_j\}_o\right] \tag{23}$$

While, the excess pore water pressure for reloading phases can be determined using the following equation:

$$\{u_j\}_{\Delta t_N} = \left[[\Phi][E_v]^{\Delta t_N}[\Phi]^{-1}\{u_j\}_{\beta_o\, t_o}\right] + \left[[\Phi][E_v]^{\Delta t_N}[D_j]^{\Delta t_N}[\Phi]^{-1}\{u_j\}_o\right] \quad (24)$$

The clay is named elastic clay if the clay remains in the same state -OC- during the time of cyclic loading. In this type of clay, it's not necessary to calculate the portion time Δt_N because the clay remain in the same state if it is subjected to either loading, constant load or unloading, no-load. The excess pore water pressure can be calculated using equations (22) and (23).

The linear average degree of consolidation can be calculated using equation (12) for loading, constant load and reloading phases. In other hand, for unloading and no-loading phases, the linear average degree of consolidation can be calculated using equation (14).

Elastic clay remains in the OC state with loading and unloading phases, so the linear settlement of layer i is calculated using the following equation according to *El Gendy* and *Hermann* (2014):

$$s_i = m_{ri}(\Delta\sigma_i - \Delta u_i)h_i \quad (25)$$

where s_i is the consolidation settlement of layer i, m_{ri} is the coefficient of volume change of layer i for unloading, no-loading and reloading, $\Delta\sigma_i$ is the increment of vertical stress at time t in a layer i due to the applied load on the surface, Δu_i is the average excess pore water pressure at time t in a layer i and h_i is the thickness height of layer i.

It should be noticed that the coefficient of volume change m_{ri} is constant and equal αm_{vi}.

Plastic clay changes from NC state to OC state or vice versa during the consolidation process. In the NC state, the linear settlement of layer i is calculated using the following equation according to *El Gendy* and *Hermann* (2014):

$$s_i = m_{vi}(\Delta\sigma_i - \Delta u_i)h_i \quad (26)$$

where m_{vi} is the coefficient of volume change of layer i for loading and constant load phases.

In OC state, the linear settlement of layer i is calculated using the following equation:

$$s_i = s_c + m_{ri}\{(\Delta\sigma_i - \Delta u_i) - (\Delta\sigma_{ci} - \Delta u_{ci})\}h_i \quad (27)$$

where s_{ci}, $\Delta\sigma_{ci}$ and Δu_{ci} are the settlement, the increment in vertical stress and the average excess pore water pressure of layer i at the pre-consolidation pressure, or at the last maximum effective stress. The coefficient of volume change m_{ri} is constant and equal αm_{vi}.

3.0 Verification of the Presented Method

3.1 Verification on Plastic Clay

Toufigh and *Ouria* (2009) performed a series of laboratory tests to investigate the consolidation of plastic clays under cyclic loading using their procedure "Virtual time method (*VTM*)". *Ouria et al.* (2015) choose two samples of them to verify their method "Disturbed state concept (*DSC*)". Sample number one in the previous reference is chosen to be verified using the *LEM*. The selected sample is a double drainage clay layer subjected to a rectangular cyclic loading with the data shown in Table 1.

Table 1 : Data of Sample number 1

q_c [kN/ m^2]	t_c [min]	H [cm]	$C_{v(NC)}$ [cm^2/ min]	$m_{v(NC)}$ [cm^2/ kN]	β [-]	α [-]
25	60	1.3845	0.0012	1.2712	0.095	0.09

The clay layer is divided into 10 layers each of 0.13845 [cm] thick, and the time is divided into 16000 intervals, each of 0.1 [Min.].

As seen in Figure 6, which shows the settlement with time for sample one, the results of the *LEM* are almost applicable to the results of the *VTM* of *Toufigh* and *Ouria* (2009). The results of the *LEM* are closer to the experimental results than the results of *DSC* of *Ouria et al.* (2015) during the first 10 cycles. Settlements with time of the different analysis methods are applicable to the experimental results. It is noted in this verification that the *LEM* achieved high efficiency in settlement calculation for this type of clays.

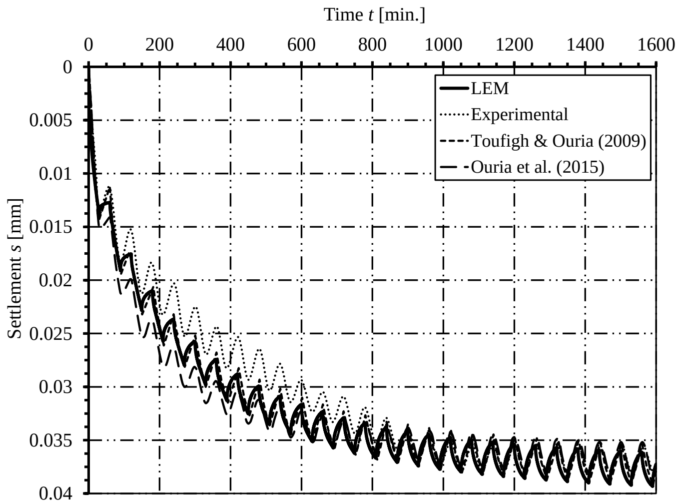

Figure 6 : Comparison of settlement s with time t

3.2 Verification on Elastic Clay

The solution developed by *Liu* and *Griffiths* (2015) was applied to a case history presented originally by *Favaretti* and *Mazzucato* (1994). The case involves the time-dependent loading and unloading with corn of a rectangular silo of dimensions 45 × 71 [m] in the town Ca` Mello near Porto Tolle, Italy. The subsoil consists of a layer of loose, silty sand overlying a thick layer of soft to very soft, silty clay. Below the clay is dense sand. In order to limit the absolute settlement and differential settlement of the silo during its lifetime, a preloading embankment wider than the silo was built on the site and maintained in place for 15 months.
The silo was constructed soon after the embankment was removed. Settlement was measured during subsequent loading and unloading cycles of the silo.

Rahalt and *Vuez* (1998) assumed double-drained conditions (total thickness of the consolidation layer, H = 15 [m]) and soft soil properties $C_v = 1.5 \times 10^{-6}$ [m^2/ sec.] and $m_v = 2.96 \times 10^{-4}$ [m^2/ kN]. Both C_v and m_v were subsequently assumed to remain constant. Owing to the silo loading, the variation of the stress with depth was assumed to be linear with maximum values at the top and bottom of the layer given by σ_t = 32 [kN/ m^2] and σ_b = 28 [kN/ m^2] respectively. *Liu* and *Griffiths* (2015) developed a computer FE program for the silo settlement problem, and also increased the loading at peak loading with 25% in both of the first two years. The results of the *LEM* for this problem is compared with the FE analysis of *Liu* and *Griffiths* (2015) and the maximum and minimum measured settlements of the silo.

The clay layer is divided into 6 layers each of 2.5 [m] thick, and the time is divided into 340 intervals, each of 0.01 [years].

As seen in Figure 7, the results of the *LEM* are identical with the results of *Liu* and *Griffiths* (2015), but smaller than the measured values in the first two years. It was also noted that the settlement of the silo reached a steady state after three filling and emptying cycles.

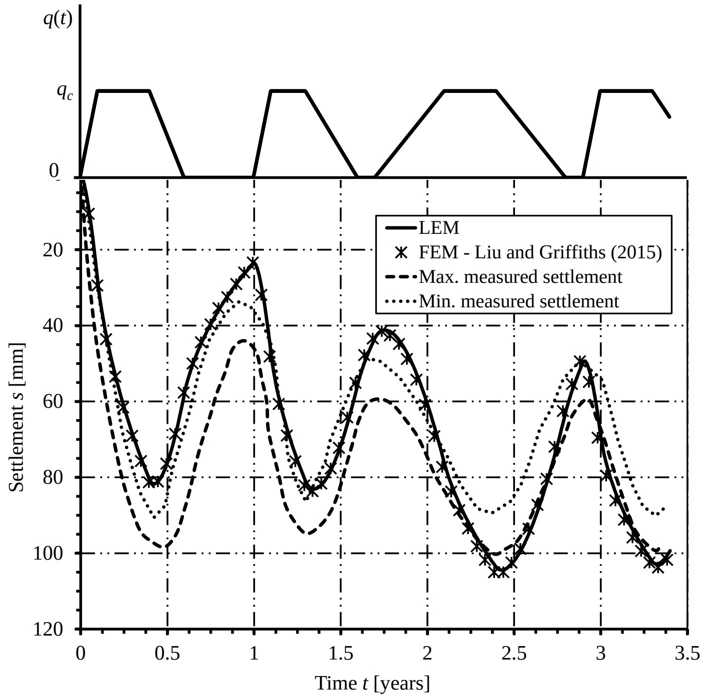

Figure 7 : Comparison of silo settlement *s* with time *t*

4.0 Application using the Presented Model

Consider that a circular storage tank has a radius of 6.5 [m] is subjected to a hydrostatic cyclic load caused by filling and discharging water. The hydrostatic load intensity is taken as 34.33 [kN/m^2]. This load is considered as circular uniform load acting on the soil profile under consideration. To represent the process of filling and discharging water in the tank, the trapezoidal cyclic load scheme shown in Figure 5. is chosen; with the load parameters t_o = 30.5 [Days], α_o = 0.008 [-] and β_o = 1.328 [-]. The tank is assumed to be constructed in two appropriate zones for the construction of circular tanks at Port-Said city in South Egypt. The geotechnical studies for Zone-1 which is located at Port-Said west were carried out by Pelli and Rossanese (2000). Zone-1 contains most of the natural gas facilities such as the condensation treatment plant owned by Petrobel Belayim Petroleum Company, Egypt. Soil investigations for Zone-2 located at the east, which contains Port-Said eastern Harbor, were carried out by the consulting engineering office. Two clay layers from the two zones are chosen to apply the LEM. The two layers are the nearest clay layers from the ground surface, one of them is OC and the other is NC. The data of the clay layers in zones 1 and 2 are shown in Table 2.

Table 2 : Properties of clay layers

Parameter	*Zone-1*	*Zone-2*
Layer thickness *H* [m]	5	2.1
Depth of the layer top below ground surface [m]	2	8.8
Overburden pressure [kN/ m^2]	22	82.6
Coefficient of volume change m_v [m^2/ kN]	0.00691	0.000773
Coefficient of consolidation C_v [m^2/ year]	1.5	0.59
Coefficient of volume change parameter α [-]	0.23	0.071
Coefficient of consolidation parameter β [-]	0.23	0.071
Bottom of the layer condition	Pervious	Pervious
Initial soil state	OC	NC

The results of the tank behavior resulting from consolidation analysis are compared for the two zones under consideration. Figure 8 shows the time variation of the average degree of consolidation for the main clay layers in zones-1 and 2. As explained before, the clay layer in Zone-1 is OC (elastic), while that in Zone-2 is NC (Plastic). The clay layer in Zone-1 is initially OC and it stay in the same state till reaching SSC after 59 cycles (2390 days). As illustrated in Figure 8, the clay layer in Zone-2 starts NC and then changes between NC and OC with filling and discharging cycles till reaching SSC after more than 60 cycles (more than 2400 days). Any further load cycling would never affect the clay layer behavior. The maximum degree of consolidation at SSC for zones-1 and 2 were found to be 81.03 and 90.66 %, respectively, while the corresponding minimum values were 62.56% and 42.370%, respectively. A maximum difference of about 54% in the average degree of consolidation was found in Zone-2 (Port Said east), where the clay is plastic.

Figure 9 shows the time-settlement curves at the center of the base. From the figure, it can be seen that the settlement is maximum in Zone-1 (Port Said west). In fact, the clay layer at Zone-1 is closer to the ground surface and it is also of a bigger height, if compared to its counterpart in the other zone. The distribution of the settlement curve for the elastic clay in Zone-1 is the same as the distribution of the average degree of consolidation with difference 23% between maximum and minimum values at SSC. On the other hand, the difference between maximum and minimum settlement values at SSC for plastic clay in Zone-2 is 3%. This small difference is related to the plastic behavior of clay.

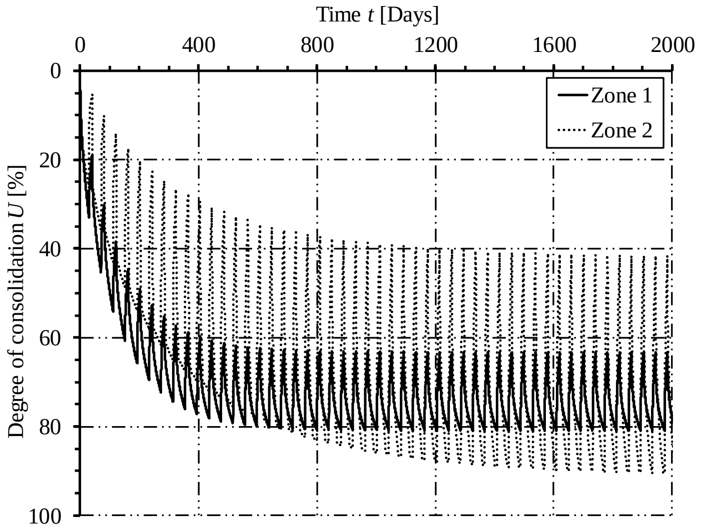

Figure 8 : Degree of consolidation U in Zones under consideration

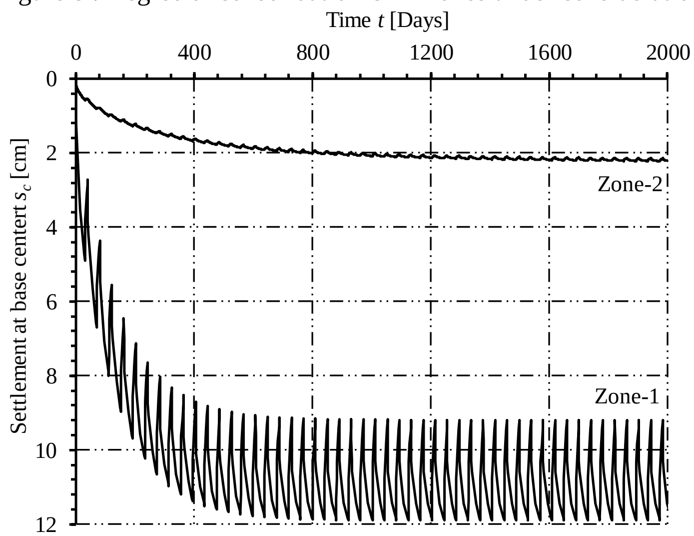

Figure 9 : Settlement s_c at the base center

5.0 Conclusion

This study is an extension of the study of El Gendy and Herrmann (2014), where they presented the LEM for analyzing consolidation of soft clay subjected to static loads. In this study, a modification is carried out to make the LEM able to analyze consolidation of soft clay subjected to cyclic loading. In the LEM, time and depth are divided into small intervals, and equations are solved numerically. The LEM is added to the commercial program ELPLA in agreement with the program authors. The LEM is applicable for normally or over consolidated multi-layered clays taking in consideration the basis of the method of Toufigh and Ouria (2009). Two verifications are chosen to test the LEM for analyzing plastic and elastic clays. The results of the verifications are in good agreement with the references results. Toufigh and Ouria (2009) and (2010) and Ouria et al. (2015) applied their methods on plastic clays subjected to initial constant stress with depth due to rectangular cyclic loading only. In this study, the LEM is applied for trapezoidal cyclic loading affecting on both elastic and plastic clay layers subjected to initial stress varying with depth. The trapezoidal cyclic load is caused by filling and discharging cycles for a circular storage tank. The data of the clay layers are chosen from real sites located at Port-Said city in Egypt. The results show that the eastern site is more suitable than the western site for constructing circular storage tanks. In future work, the LEM can be developed to be able to analyze any distribution of time-dependent loading with more realistic soil parameters.

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